Precast Bulb Tees Made Integral with Substructure – The Ellis Vieser Memorial Bridge

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ABSTRACT

The innovative use of precast Bulb T's made integral with the substructure for the approach structures proved to be the decisive edge for the contractor in their winning bid on this $60 million Design/Build bridge project. Structure type selection was left open to the bidders for this 4,288-ft. long bridge which carries 3 lanes of traffic plus shoulders. With a wide range of required span lengths (133-9" to 173'-0") for the approaches due to a mandate to match the existing bridge pier locations, a versatile system was needed. Additionally, the need for tall piers to provide a minimum of 108 feet of vertical navigation clearance affected the type selection. The winning design for the approaches was based on a concept developed and tested in California for precast girders in areas of high seismicity. Longitudinal bracing of the substructure provided by a moment-resisting connection to the superstructure reduced the demands on the piers and foundations thereby improving the structural performance and reducing construction cost.

Keywords:

Spliced Girders, Integral Bents, Continuous Bridges, Bulb Tee Girders, Post-tensioning, Bent Caps, Seismic Design

INTRODUCTION

The Raritan River is an important transportation node in the New Jersey highway system. Bounded by Perth Amboy on the North and Sayreville on the South, the river is crossed by the Garden State Parkway (Driscoll Bridge – soon to be 16 lanes), US Route 35 (Victory Bridge – 4 lanes) and US Route 9 (6 lanes). Six miles away, 12 lanes of the NJ Turnpike (I-95) traverse the river. See Figure 1 and Photo 1.

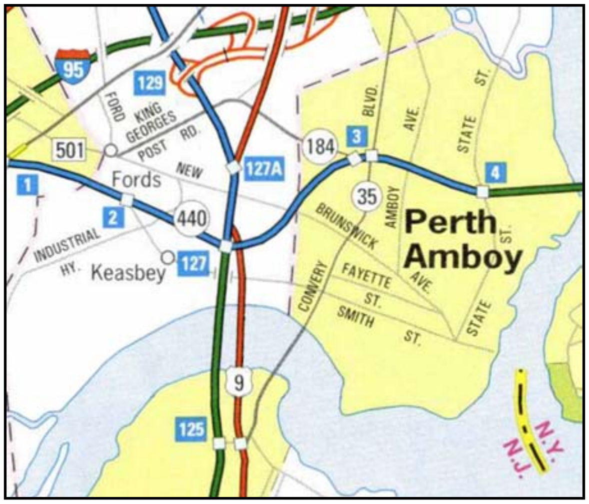

Figure 1. Site Map

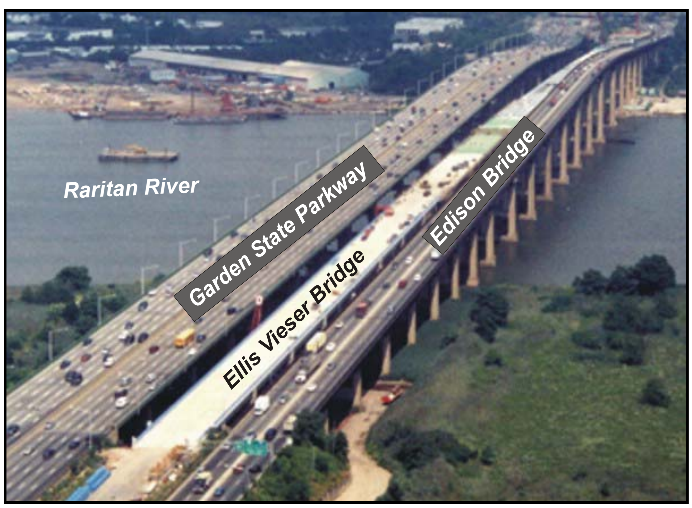

Photo 1. View of Area Bridges
(Looking North)

The existing Edison Bridge originally carried two lanes of traffic in each direction without shoulders. NJDOT advertised a Design/Build project in February, 1999 to provide a new twin structure to carry three lanes of one-directional traffic with shoulders in the Southbound direction. After completion and after all traffic was transferred to the new bridge, the existing

Edison Bridge had a new superstructure constructed that would carry three lanes plus shoulders in the Northbound direction. After completion in 2002, the new SB bridge was named the Ellis Vieser Memorial Bridge.

Being a Design/Build bid, structure type selection was left open to the contractors for this 4,288-ft. long bridge. With a wide range of required span lengths (133'-9" to 173'-0") due to a mandate to match the existing bridge pier locations, a versatile system was needed – see Figure 2.

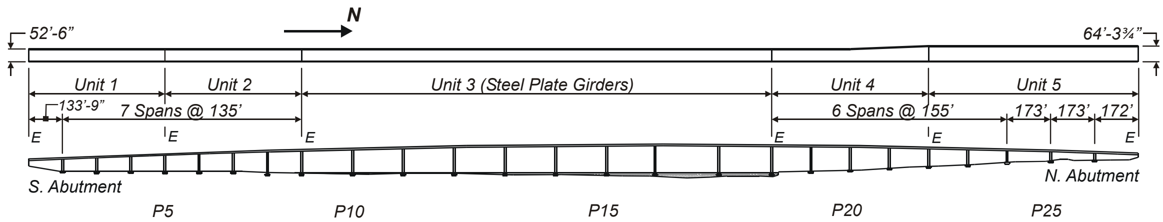

Figure 2. Plan & Elevation of Bridge

Additionally, the need for tall piers to provide a minimum of 108 feet of vertical navigation clearance affected the type selection. The winning design for the approaches was based on a concept developed and tested in California for precast girders in areas of high seismicity.[1,2] While the seismic demands for the project were moderate (SPC "B" with Acceleration Coefficient = 0.18), the chosen system provided and excellent longitudinal bracing of the substructure. This was accomplished by a moment-resisting connection of the superstructure to the substructure. The use of the system reduced the demands on the piers and foundations thereby improving the structural performance and reducing construction cost.

After evaluating many structural systems/schemes, the contractor chose a design for the approaches based on 4 and 5 span continuous bulb tee girders made integral with the substructure - Units 1, 2 , 4 and 5. This paper will discuss:

- The development of the system based on moment continuity connection of the girders to the substructure;
- Selection of splice location and framing scheme based on span length required;
- The staged construction of the superstructure including post-tensioning;
- Computer modeling and staged analysis with results presented graphically;
- Comparison of time-dependent concrete stresses based on age of girder and different modulus, creep and shrinkage codes

The cross-sections for the chosen system are presented in Figure 3.

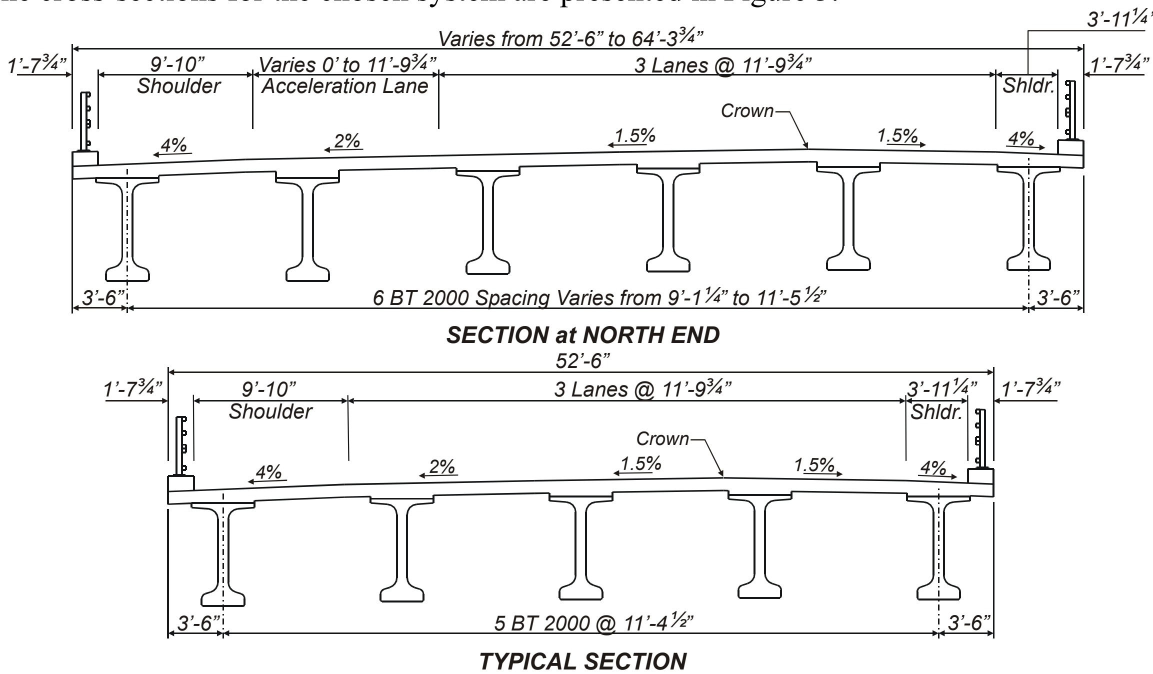

Figure 3. Bridge Cross-Sections

SYSTEM DEVELOPMENT

Over the course of the late 1980's and into the '90's, California saw a decrease in the number of bridges being built with precast concrete girders. What had once been a strong market now saw less than a 3% market share. The market had principally been reduced to new girders for widenings of existing bridges built in the 1960's and 1970's. With the reactivation of the Precast Concrete Manufacturers' Association of California (PCMAC), the precast industry sought to address the declining market share issue.

Starting in 1992, PCMAC developed an initiative to revive the precast bridge girder market. Working with Caltrans, the authors developed concepts that would return precast girders to a state of competitiveness with the typical California cast-in-place, post-tensioned box girder bridge[4]. Studies indicated that the precast girder bridge form had not kept pace with the increasingly stringent requirements placed on structures in areas of high seismicity. Precast girders were still being used as simple girders supported on bearings, which in turn were supported on drop cap bents – a system that is very inefficient in resisting high lateral forces generated by earthquakes - particularly in the longitudinal direction. CIP box girders had a distinct advantage in the reduced foundation demands due to the integral connection of the superstructure and substructure.

A system needed to be developed that made the superstructure continuous and furthermore, made the superstructure continuous with the substructure. In addition, the span range for the precast girders had to be increased from its then 125-ft. limitation to the 200-ft. range in order to enhance its appearance as well as to meet the needs of the California market. Since a new system would require new shapes and forms, full advantage could be taken of the advancements made around the country in precast girder technology and efficiency. The system developed was based on the following:

- A basic girder shape patterned after the new Bulb Tee girder shapes, modified to contain post-tensioning ducts for superstructure continuity;
- For long spans, the use of a pier segment that is continuous over the support thereby eliminating complicated splicing details in an area of maximum stress;
- An integral connection of the precast girders to the columns through the use of a simple, cast-in-place, post-tensioned diaphragm;
- Splicing details using post-tensioning that provide continuity of the superstructure with spans up to 180-ft. for a 72” deep girder and over 200-ft. for a 81” deep girder.

The last step of the development process was the testing of the system to demonstrate the correlation between theoretical and actual performance. Caltrans and PCMAC sponsored the testing program and the project was undertaken by UCSD.

TESTING OF A SPLICED PRECAST GIRDER BRIDGE [1, 2]

Introduction

The research program to construct and test two 40% full scale models under fully reversed longitudinal seismic loading was undertaken at the University of California, San Diego (UCSD). Design, construction and testing of the first model, which utilized Modified Florida-Bulb-Tee girders, was completed in June 1996. The second test, conducted in 1997, was of similar scale and incorporated “U” or “Bath-tub” girders. The testing program at UCSD not only verified the structural adequacy of the integral column-superstructure details under simulated seismic loads, it also allowed engineers to evaluate the constructability of these details via large scale models in the laboratory.

The main focus of this research was to study the effects of longitudinal seismic forces on the column-superstructure continuity. The prototype structure (Figure 4) consisted of four 160-ft. spans and a single-column. The model test unit dimensions and forces were scaled directly from the prototype structure. The region selected for study included the column, bent cap and the full width superstructure extending from midspan to midspan.

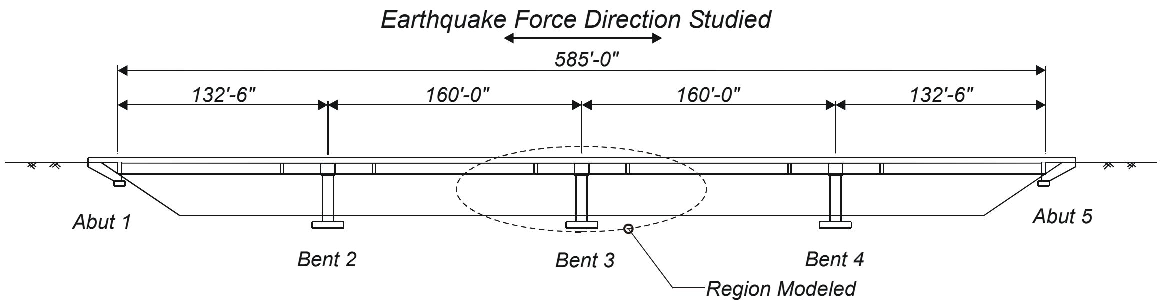

Figure 4A. Prototype Bridge Layout

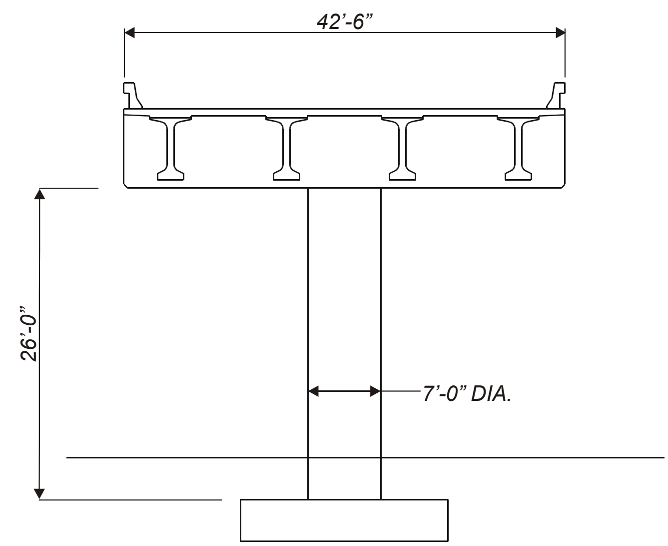

Figure 4B. Prototype Bridge Cross-Section

Load was applied to the model through two horizontal actuators placed on both sides of the unit to model the seismic inertia forces acting along the bridge under longitudinal response. The four vertical actuators located at the corners of the test unit applied the seismic shear to the girders. Figure 5 presents the testing setup.

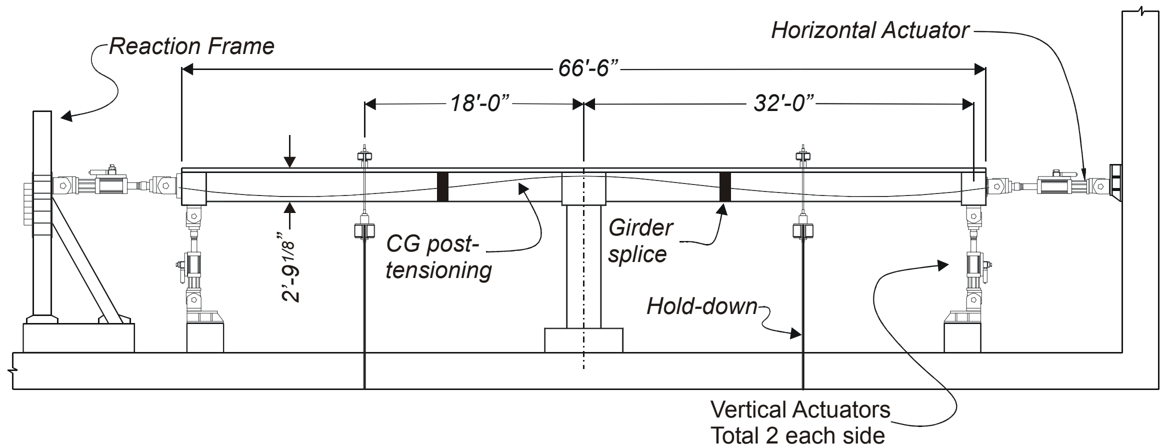

Figure 5. Schematic of Test Set-Up

Design

Another important aspect of the testing program was the "proofing" of the design approach. Key issues included (1) the resistance of girders as a function of distance from the columns, (2) the transfer of forces in the development of the plastic moment at the top of the column, (3) the torsional demands on the cap, and (4) the joint steel requirement at the cap/column interface. These issues were addressed by UCSD from both the theoretical and actual perspectives. Lessons learned from the testing enabled UCSD to adjust their design methodology. For a detailed treatment of the system design issues, see References No. 1 and 2.

Test Results

Columns: Testing of the first model was successful under simulated seismic loads. Ductile plastic hinges formed at the top and bottom of the column with little strength degradation up to a displacement ductility of eight (μ_{Δ} = 8) which was well beyond the design-ductility capacity of four (μ_{Δ} = 4). The structural displacement ductility (μ_{Δ}) is defined as the ratio of displacement to idealized yield displacement. Confinement hoops in the plastic hinge region had fractured at μ_{Δ} = 8, which indicated the onset of plastic hinge failure.

Superstructure: The superstructure performed essentially elastically under simulated longitudinal seismic response with only minor cracking observed. Due to prestressing, the cracking in the bent cap and the girders closed up upon removal of the seismic loads, making potential repair of the superstructure after a design level earthquake essentially cosmetic. For a more detailed description of the testing program, see Reference No. 1 and 2.

THE PRECAST SYSTEM

The newly developed and tested system that addressed the requirements of superstructure and substructure continuity, aesthetics and minimization of traffic impact during construction was introduced in California in the late 1990's. A cross-section for both single and two-column bents is shown in Figure 6.

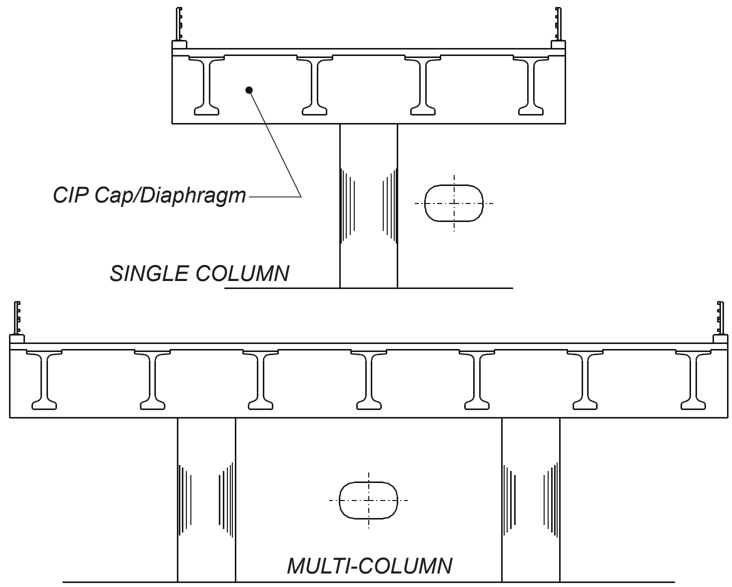

Figure 6. Single & Multi-Column Applications

For maximum flexibility in locating piers, the system employs two types of splices. Where girders can be shipped in lengths equaling the required span lengths, the splice may be located over the pier. Where a longer span is required, i.e., one that cannot be shipped as a single piece, the splice may be located in the span at the approximate inflection point. The superstructure of this bridge system consists of three basic components as shown in Figure 7.

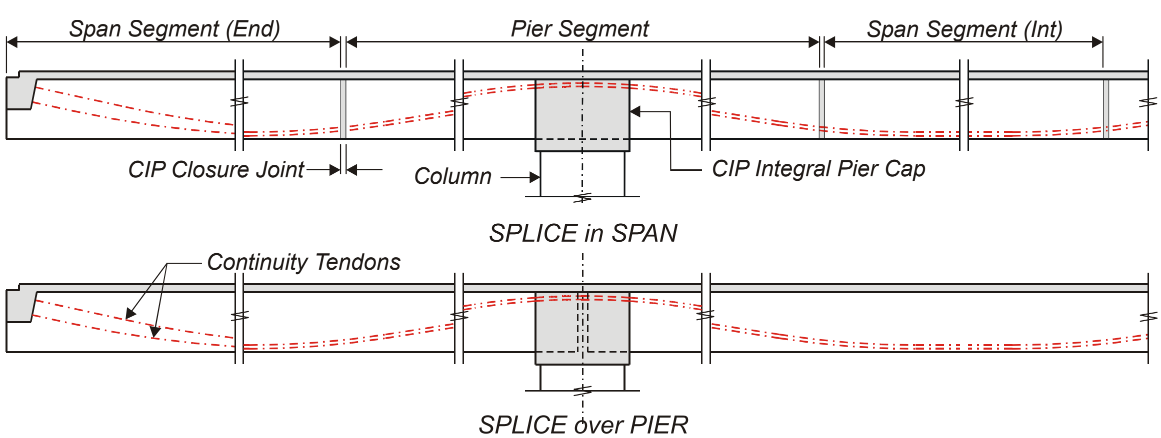

Figure 7. System Elevation

An important feature of the system is the use of a two staged continuity post-tensioning. The first stage consists of stressing the "girder only" system after the integral caps and splices had been formed, poured and stressed. The second stage consists of stressing the composite system after the deck has been poured and cured to an appropriate strength. The benefit of the two stage post-tensioning is the reduction in pre-tensioning demands because the girders resist the dead load of the wet slab as a continuous girder.

Pier Segment (Precast): For the Splice in Span System, the pier segment is a variable length precast section comprised of a prismatic bulb tee section. The length is variable in order to locate the splice at the approximate point of inflection for a given span arrangement. Subject to weight limitations, the web in the central portion of the pier segment may be thickened to accommodate the large negative moments over the pier. The section is pretensioned for shipping and handling stresses and for a portion of the service negative moment over the pier. The Pier Segment also contains ducts for two stages of longitudinal post-tensioning: one for the girder-only section and one for girder-deck composite section.

Span Segment (Precast): This drop-in section traverses the positive moment region of a given span and utilizes a standard bulb tee shape. It is pretensioned for lifting and handling stresses and contains ducts for the two-stage post-tensioning of the continuous girder and composite sections.

Pier Diaphragm (Cast-in-Place): This portion of the system provides the connection of the precast Pier Segment to the column as shown in Figure 8. This represents the diaphragm for Pier 19 of the Ellis Vieser Bridge Project. The pier is a two column, rigid frame bent. The Pier Diaphragm is formed and poured around and under the Pier Segment. The entire system is unified by means of transverse post-tensioning through the Pier Segment. Reinforcing steel in the top slab and in the diaphragm below the Pier Segment improves the monolithic response of the superstructure-column interface. The principal mechanism for developing the monolithic response is a combination of torsion and shear-friction through the Pier Diaphragm (integral bent cap), which then translates into longitudinal bending of the girders.

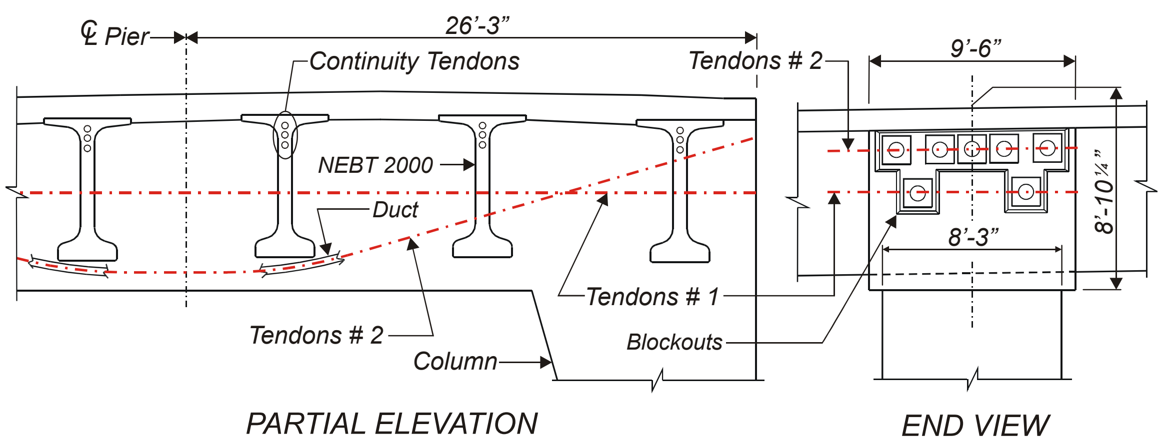

Figure 8. CIP Pier Diaphragm

One of the important features of the integral system is its minimal impact on traffic during the construction process, compared to CIP box girder systems. This is of critical interest in regions where bridge construction occurs in dense, urban areas with minimum vertical clearances. A typical construction sequence for the system is depicted in Figure 9.

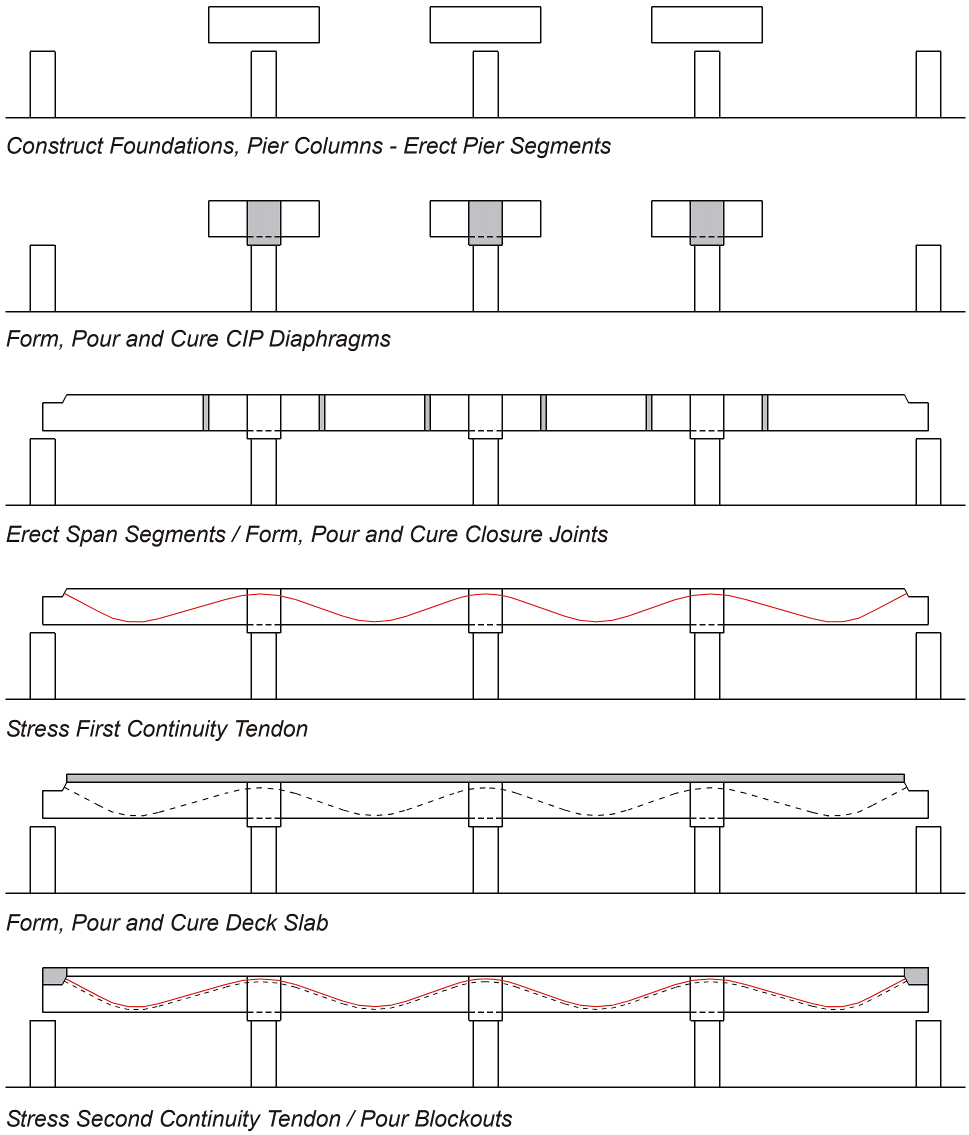

Figure 9. Typical Erection Sequence

Photo 2 shows the temporary support that was used on the Ellis Vieser Project to support the pier segments while the diaphragm was formed and poured. Photo 3 shows the strongbacks used to support the span (drop-in) girders prior to forming and pouring the closure joint between the pier segment and span segment.

Photo 2. Temporary Girder Support at Pier

Photo 3. Strongbacks Supporting Span Segments

Integral Substructure Design

The most common form of support for typical precast/prestressed girder bridges is the multi-column bent consisting of circular (or rectangular) columns with a rectangular bent cap. The columns, in turn, are supported on either isolated or combined footings. In California, a common bridge type consists of cast-in-place prestressed concrete box girders monolithically connected to the substructure to create longitudinal frames with multiple spans. The box girders are typically constructed on falsework, and in many cases, can be supported on a single column substructure. Multi-column bents are provided for wider bridges. Unlike the precast girder system of a drop cap pier, the CIP box girder system with single-column supports resists longitudinal forces in double curvature bending. This is a decided advantage in areas where large longitudinal forces are possible as from a seismic event. Emulating such a response in a precast/prestressed girder system would restore the competitive edge to precast concrete while eliminating the need for the extensive shoring commonly required for CIP construction.

The lack of monolithic action between the superstructure and bent cap in typical precast/prestressed girder systems causes the column tops to act as pins. Consequently, while the transverse stability of multi-column bents is ensured by frame action in that direction, stability in the longitudinal direction requires the column bases to be fixed to the foundation supports. This requirement places substantial demands on the foundations of multi-column bents, particularly in areas of moderate to high seismicity. Developing a moment connection between the superstructure and substructure makes it possible to introduce a pinned connection at the column bases, and thus require less expensive foundations. At the very least, a reduction in moment will occur by the change from cantilever bending to double curvature bending in the columns.

Integral bent caps are also beneficial in precast/prestressed girder systems with single-column bents. In these systems, the longitudinal seismic forces are typically resisted through single-curvature bending of the columns (i.e., the columns act as cantilevers). Seismic forces are generally engaged by a pintle system. By introducing moment continuity at the cap levels, the columns are forced into double-curvature bending, which tends to substantially reduce the moment demands. As a result, the sizes and overall cost of the foundations are also reduced. The concept is presented in Figure 10.

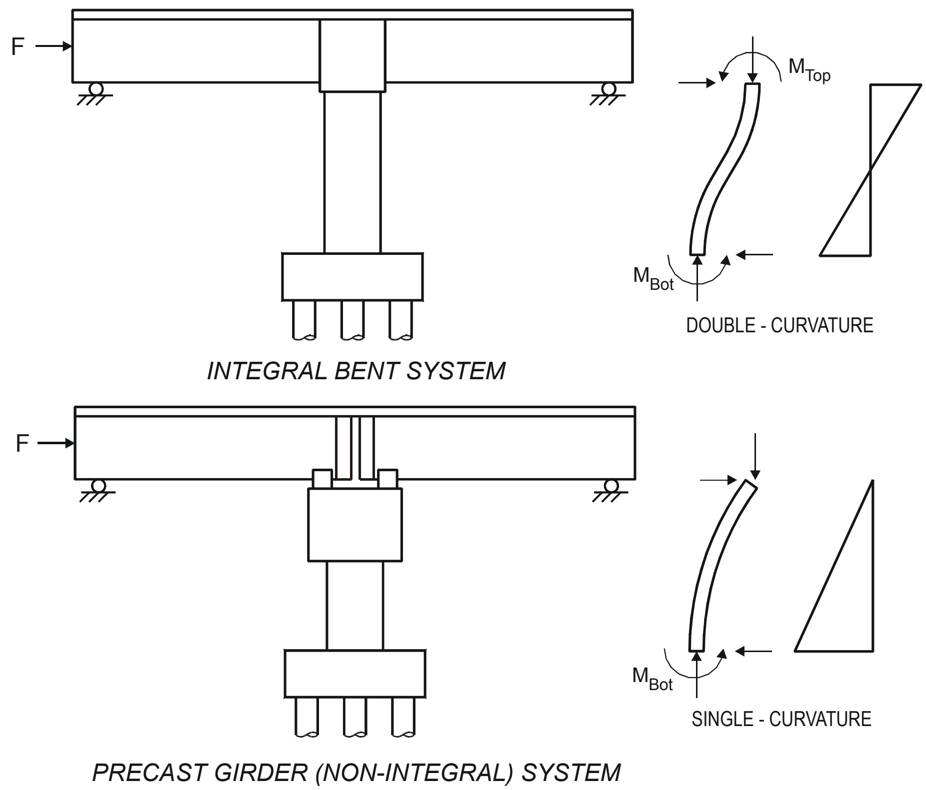

Figure 10. Single & Double Curvature Column Bending

There is however a tradeoff in that the integral system will increase the frequency of the structure thereby reducing the period. This will shift the structure to the left on the Spectral Acceleration curve which will result in a higher acceleration and therefore seismic force. The other demand that is created on the column is that resulting from live load. With the integral connection, live load bending moments will now be introduced into the columns and foundations. The advantage of the double curvature bending is generally so large that the increased seismic force and live load bending can be accommodated and the system still be more economical than the single curvature bending system.

THE PROJECT

GENERAL

The project required that the piers of the new bridge match the spacing of the existing Edison Bridge. This necessitated a range of spans from 133'-9" to 173'-0". Shipping length and weight were a consideration as the chosen girder segments were permitted for weight and escorted for length. Above 100-ft. in length, two escorts were required - one front and one rear. The length limitation of 135-ft. was governed by transportation issues such as turning radii along the delivery route. Photo 4 depicts the truck transport system.

Photo 4. Delivery of Girder

Accordingly, girder splices were determined by the length of transportable girder. Spans less than 135-ft. were spliced over the piers while spans greater than 135-ft. were spliced in the span at the approximate inflection point. The approach piers ranged in height from 50-ft. to 125-ft. as measured from top of footing to CG of the diaphragm. Bracing effects in the longitudinal direction became a consideration. The typical column section is 7'-3" x 8'-3" with the 8'-3" dimension in the longitudinal direction. Photo 5 presents the typical pier shape prior to setting of the girders. The shape of the pier was dictated by the constraint that the architecture of the new piers be similar to the existing piers.

Photo 5. Pier Construction Looking North

The project was designed in accordance with AASHTO LFD Specifications (1998) as amended by NJDOT. The creep and shrinkage parameters were taken from the CEB-FIP Model Code for Concrete Structures, 1978 Edition. The Design/Build specifications required a minimum of five girder lines. With a maximum girder spacing of 11'- 5 1/2", NJDOT required a deck thickness of 9 3/8". The deck, girder and cap concrete was specified as 6,000 psi while the column concrete was specified as 4,000 psi. The allowable service load tension for the superstructure design was $7.5 \sqrt{f'_c}$.

Given the requirements and constraints of the project, the authors believed that the integral, continuous girder system developed for areas of high seismicity would be a good solution due to its bracing capability and flexibility of span length.

STATICAL SCHEME

After evaluating several options for the approaches including steel plate girders and segmental box girders, the economics of the bulb tee scheme dictated its selection. With relatively tall piers, the longitudinal bracing and double curvature column bending offered by the integral framing resulted in an appreciable reduction in demand on the foundations. The integral connection also reduced positive live load demands on the girders. Table 1 presents a comparison of the live load moments for the girders in Unit 5 based on different statical schemes.

System	End Span	Interior Span
Simple Girder	2,487	3,074
Continuous	2,071	2,043
Continuous w/Integral Framing	1,798	1,579

Table 1. Positive Live Load Moments (k-ft)

The effect of longitudinal bracing on the piers is seen by observing the results of the analysis based on two statical schemes, i.e., pinned at the top and integral at the top. Table 2 presents the data for the non-integral and integral schemes at top of footing for the un-factored longitudinal seismic load case.

Statical Scheme	Period (sec)	Cs (g) (g)	Seismic Moment per Column at Top Ftg (k-ft)				Seismic Shear per Column at Top Ftg (k)			
			Pier Number				Pier Number			
			23	24	25	26	23	24	25	26
Integral Top	1.22	0.19	9,491	14,640	6,314	18,650	552	1,033	663	1,813
Pinned Top	1.76	0.15	10,610	16,490	13,080	28,490	319	622	598	1,638
% Integral			112%	113%	207%	153%	58%	60%	90%	90%

Table 2. Un-Factored Seismic Foundation Design Forces

PIER DIAPHRAGM (INTEGRAL CAP) DESIGN

The pier diaphragm serves two primary functions: (1) it carries gravity loads from the superstructure to the columns, and (2) it acts as a seismic connection between the superstructure and substructure. Designing the pier diaphragm to satisfy the first function is straightforward and could be accomplished using any available computer program capable of post-tensioned concrete design.

However, designing for the second function requires a procedure that has evolved from the successful experimental testing of the Florida Bulb Tee girder, and other research, at the University of California at San Diego. Details of the procedure can be found in References 2 and 5.

The goal of a seismic connection is to transfer the top-of-column plastic demands into the superstructure without yielding either the connection itself or the girder ends. To achieve this, both the connection and the girder ends have to be designed to provide ultimate strength resistance levels (capacity) exceeding those of the forces transferred (demand). Additionally, the connection should be detailed to ensure adequate distribution of the longitudinal moment from the top of the column to the various girders.

The main steps of the design procedure are outlined below:

1. Determination of the column top and bottom plastic moment capacities. This is typically a function of the concrete column section shape, reinforcement, material properties, and axial load. Where seismic design is not required, the factored moments and axial loads at the top of column should be used instead.

2. Calculation of the principal stresses in the bent cap due to joint shear. The "deflection" of the column forces into girder forces is accompanied by significant shear stresses within the joint region connecting the column and girder. The forces and principal stresses in the joint could be calculated from statics, based on simplifying assumptions regarding the effective areas over which the forces act. The magnitude of the principal tensile stress in the joint is used to determine the reinforcement requirements of the joint as follows:

    - If principal tension stress $\leq 3.5\sqrt{f'_c}$, no vertical joint reinforcement is needed, and only nominal transverse reinforcement is required.
    - If principal tension stress $> 5\sqrt{f'_c}$, all requirements for joint reinforcement (Step 3 below) must be met.
    - If principal tension stress is between $3.5\sqrt{f'_c}$ and $5\sqrt{f'_c}$, linear interpolation between full and nominal requirements for joint reinforcement must be met.

3. Design of joint reinforcement, if required per step 2, is performed in accordance with the procedure described in References 2 and 5.

4. A Torsion-Shear friction analysis to verify the bent cap's ability to transfer the column plastic moments to the bridge superstructure. In the absence of the bottom slab as in the case of the bulb tee section, column moments and shears are transferred into the girders completely through torsional mechanisms. Due to the limited length available between the face of the column and the girder, spiral cracks typically associated with torsion cannot fully develop. Therefore, conventional torsion design methodologies that are primarily based on this cracking pattern are not applicable. Instead, the torsional capacity is calculated in terms of the plastic friction model as described in References 2 & 5.

5. A check of the bridge superstructure capacity to ensure that the plastic hinges form in the column rather than the superstructure. The bulb tee girders are considered protected elements, and hence, are required to remain elastic throughout a seismic event. Allowing plastic hinges to form in the girders near the supports will likely be accompanied by a loss of capacity against vertical loads, and could potentially lead to bridge collapse.

GIRDER DESIGN & COMPUTER ANALYSIS

One of the principal issues when designing precast girder systems made continuous is the age of the girder when erected and stressed. The age affects the creep as well as the shrinkage of the girder. In a simple beam application, creep and shrinkage reduce the prestress force and cause displacement in the girder. In a continuous system, in addition to the above effects, non-beneficial secondary moments will be generated. Another consideration in composite structures is the age differential between the slab and the girder. Since the slab is cast-in-place, it’s shrinkage will induce positive moments in the beams. This in turn will generate negative resisting moments at the piers. The bigger the age difference between the slab and girder then the larger will be the shrinkage induced moments in the system.

The key parameter affecting the amount of creep is the age of the concrete when loaded. It is obviously beneficial to wait as long as possible prior to erecting a girder but that generally is not possible in a real world environment. Similarly with shrinkage, the longer a girder remains in storage, the more shrinkage will be taken out of the girder.

Table 3 presents results from comparative computer analyses using BDII software[3] where the only difference between the runs was the age of the girder when erected. It presents a good picture of the effects of girder age on creep and shrinkage. The results shown are extreme fiber stresses – bottom for in-span sections and at the top for the section over the first interior pier.

Age When Erected	Long Term Extreme Fiber Stresses (psi)		
	0.4 PT End Span	1st Interior Pier	0.5 PT 1st Interior Span
30	1,265	484	935
60	1,303	501	985
90	1,331	516	1,027
120	1,348	530	1,057
180	1,381	560	1,113

Table 3. Parametric Study of Age of Girder Effects

While the beneficial part of increased age comes from reduced losses in the post-tensioning, a detrimental part comes from the secondary moments generated by creep and shrinkage. Figure 11 presents a plot of the moments generated by creep and shrinkage as a function of the age of the girder when spliced.

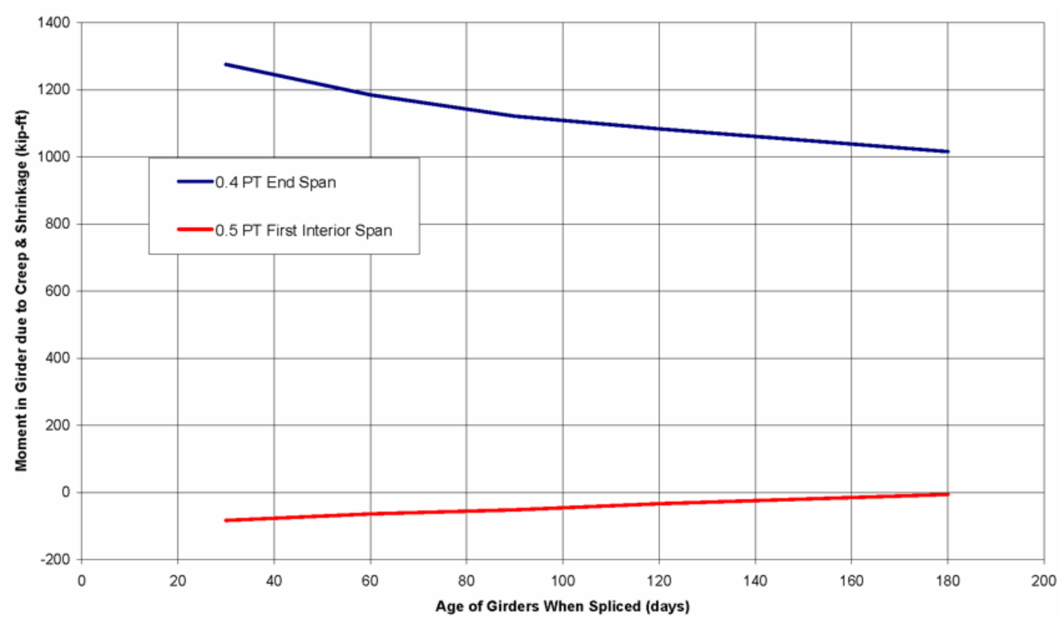

Figure 11. Creep & Shrinkage Moment Variation with Age of Beam

The age of girder issue needs to be resolved early in the process particularly if it is being decided in a Design/Build environment. Once the project schedule is determined a casting and erection date can be established. For Design/Build projects, one must remember that the design must be approved prior to casting. While the design production and approval process is going on, the contractor will be constructing foundations and substructure. There will be pressure to cast girders once the design is approved so typically, younger ages of girders are assumed in the design. Given the impacts to the design of girder age, it is safer to assume a younger age. For the Ellis Vieser Bridge project, the age at which the girders were made continuous was assumed to be 44 days.

Figure 12 presents a parametric study of positive and negative moment based on various codes available for design. These relate to the parameters of modulus of elasticity, creep and shrinkage and how these parameters vary with respect to time. For two points on the bridge, one in a positive and the other in a negative moment area, the bottom and top fiber stresses are plotted as a function of time. Three curves are presented – one each for parameters based on the CEB-FIP Code 1978, CEB- FIP Code 1990 and the ACI Code (209 – 1997).

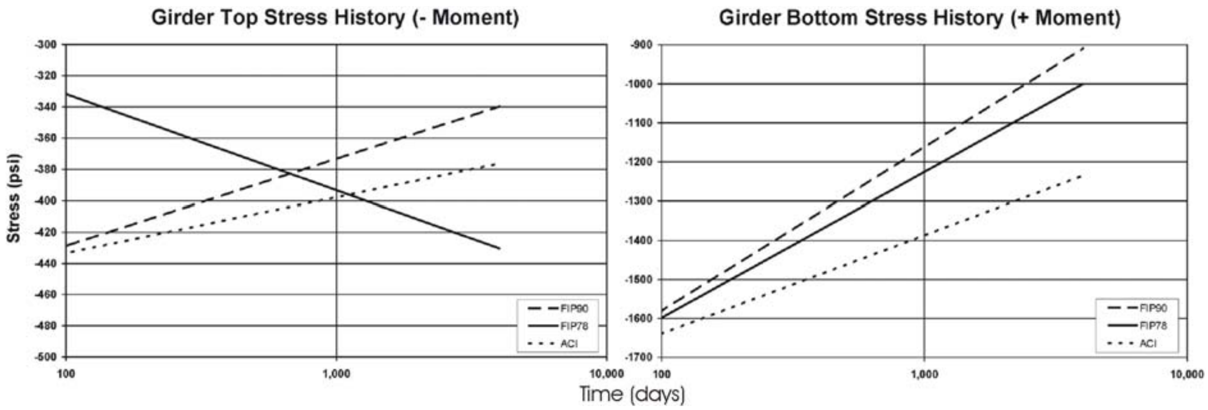

Figure 12. Moment Comparison for Different Codes

These plots show that the various modulus, creep and shrinkage model codes result in variations that may be as much as 25%, or more. Experience has shown that time-dependent effects are best treated as limits of behavior (i.e., envelopes) rather than precise predictions. Another variation between codes is in the time history of the development of the creep and shrinkage strains. So while reasonable agreement may be obtained for the ultimate strains, agreement may diverge at times prior to attaining the ultimate strain.

Both high and low estimates of these effects should be accommodated in the design for service conditions. For example, bearings, bearing offsets, expansion seats and joints should be sized to accommodate high and low estimates of the time depended movements, as opposed to any specific calculated values. Failure to do so could sometimes result in unacceptable service conditions due the discrepancy between actual and predicted behavior.

CONSTRUCTION

The construction went exceptionally well. The only significant issue was a conflict of the column steel with the diaphragm steel and post-tensioning. It was resolved on the first diaphragm cast and was not an issue thereafter. There was a requirement to match the architecture of the existing bridge which was to remain. Since it was a two girder system with floorbeams, the columns were located under the girders. Accordingly, a precast girder was required over each column which gave rise to a potential for conflict. In a more typical project, the columns would be located between the precast girders.

Some representative construction photographs include:

Photo 6. Erecting Precast Pier Segment

Photo 7. Splicing Girders over Piers 6 & 7

Photo 8. Splicing Girders at a Pier

Photo 9. Construction at North End
(Looking South West)

CONCLUSIONS

The Ellis Vieser Bridge carrying Rt. 9 SB over the Raritan River showed that precast concrete construction is adaptive to the design/build environment. The adaptation from a system developed for use in areas of high seismicity proved to be the deciding factor in the contractor's selection of precast girders over steel plate girders. When studying a system for a new bridge project, the integral system should be judged in its entirety including the forming and casting of the integral diaphragm as well as savings to be realized in other areas.

The benefits and therefore the savings of the system include:

- Reduction of substructure demands due to the creation of longitudinal frames; this permits precast girder systems to be competitive with CIP box girder systems in areas of high seismicity. This also permits reduced column dimensioning.
- Reduced live load demands on the precast girders due to integral framing with substructure.
- The girders and slab are precompressed for service loads which are resisted with an uncracked section thereby providing improved durability.
- Elimination of bearings and future maintenance concerns at integrally framed piers.
- Elimination of temporary in-span falsework towers at the splice locations. The moment connection between the cap and pier segments provide the necessary stability to support the weight of the drop-in segments.
- Elimination of large drop caps which impact the aesthetics of the final structure.

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